

Structural Performance of Cold Formed Section Beam to Column bolted Connections.

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Abstract

The semi-rigid nature of the cold formed section beam to column connections is primarily due to the distortion, tearing of column section plates, and distortion of beams end sections. Stability of such structures usually depends significantly on the behavior of these connections. In this study, Nonlinear finite element model were done for two cold formed section assemblies to evaluate their rotational stiffness and capacity. In the first assembly, both the beam and column having a back to back lipped sigma cold formed sections. However, in the second assembly, lipped back to back channel section connected to box column section. The beams and columns are connected together using gusset plate or gusset plate in-addition to angles between the beam and column flanges. In the model, the column ends were fixed, and the beam cantilever end allowed to move vertically. The out of plane deformation were prevented by means of lateral restraints attached to the specimens at the tip of the horizontal beam below the load application, and at the mid span of the cantilever beam. The cold-formed sections were modeled using shell elements while the connecting fasteners are modeled using contact elements. The developed model allows for the geometric as well as the material nonlinear behaviour of the assembly. An extended parametric study was then carried out covering the different parameters such as thickness of the main sections and the connecting elements. Finally, the numerical M- θ curves were drawn and the different modes of failure were explored. Results reveal that the connection could reach the flexural capacity of the connecting elements but with large deformations.

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1. Introduction

Recently there are rapid growth in employing cold formed steel sections in steel construction industry, due to its high strength-weight ratios, fast time erection, and durability. However, the main problem with the cold formed section is its slenderness because it has very thin thicknesses relative to its width so that several failure modes can occur such as local, distortional, and global buckling. Consequently, connections of cold formed sections cannot reach its full rigidity like hot rolled sections.

Strength of beam to column connections attracts the attention of several researches. R.F. Pedreschi et.al. [1] describe and summarize the application of a new technique known as press joining. They tested series of full scale trusses using this type of connections. They concluded that the shear strength of press joints depends on the direction of the applied loads. In addition, connections using multiple press joints can resist bending moments. H.B. Blum, K.J.R. Rasmussen [2,3] conduct numerical and experimental parametric studies on a hunched portal frame composed of back-to-back lipped channel sections bolted through the webs for the main frame members, and back-to-back L-brackets bolted through the webs at the connection. They found that the knee to column connection is the critical element in the frame, and modifications to this connection can increase the frame ultimate vertical load by 25% for applied gravity loads and 37% for applied wind and gravity loads. B.Tshuma, M. Dundu [4] tested two different configurations of connecting single channel cold formed rafters used in double bay portal frames. They demonstrated that these connections were not critical. Teofil-Florin et.al [5] described numerical and experimental results for eaves and apex connections using back to back KB600-5 and KB450-3,5 profiles. They concluded that local buckling of the web cold formed steel profile and bolt holes elongations in the tension zone of connection plate prevents the system from reaching its expected true capacity. Jun Ye, et.al., [6] investigate the efficiency of bolt friction-slip mechanism in improving the seismic performance of CFS moment resisting connections. They found that, using bolting friction-slip mechanism can significantly improve the energy dissipation capacity of the connections. In addition bolt configuration and CFS beam cross-sectional shape and classification play the main roles in the CFS moment connections. Wei Chen, et.al., [7] observed that the shear strength of cold-formed steel-to-steel screwed connections at ambient and elevated temperature could be conservatively predicted based on the AISI S100-16 and using the reduced material properties of the corresponding cold-formed steel.

2. Case of Study

In this study two cases are examined. The first one which is named as case1, the beam and column cross sections are back to back sigma sections. While in case 2, the columns have box cross sections, and beams have back to back lipped channel cross sections. Geometric dimensions of case 1 and case 2 are illustrated in Figure1 and Figure2; respectively. In case 1, the beams and columns are connected mainly by tapered gusset plates with variable thickness, t_g , that ranges from 2mm to 10mm. The gusset plates are connected to the column web by 24 bolts having diameter of 12mm. The spacing between bolts are 100mm in the vertical and horizontal directions. Note, this bolt arrangement is fixed on all the studied cases. However, beams web are connected to the gusset plates by variable number of bolts that ranges from 4 to 12. Spacing between bolts are also 100mm in the vertical and horizontal directions. The ultimate capacities of 11 connection configurations of this case are determined. The parameters studied are thickness of gusset plate, t_g , number of bolts between gusset plate and beam web, adding angle between beam and column flanges, and finally presence of horizontal stiffeners in the column. Details of these 11 connections are listed in Table 1. In case 2, Folded gusset plates are used to connect beams and columns. Thicknesses of gusset plates are 2mm, and 3mm. This plates are connected to the sides of box sections by two rows of a single hex-washer head, 5 mm diameter screw near the top and bottom edge of the gusset plate. Spacing between screws is 30mm. However, beam webs are connected to the gusset plates by bolts with diameter 12 mm. Eight configurations of this detail as illustrated in Figure 2 are examined. Note that, bolts that are used in connections having $F_u = 460\text{MPa}$, $F_y = 270\text{MPa}$.

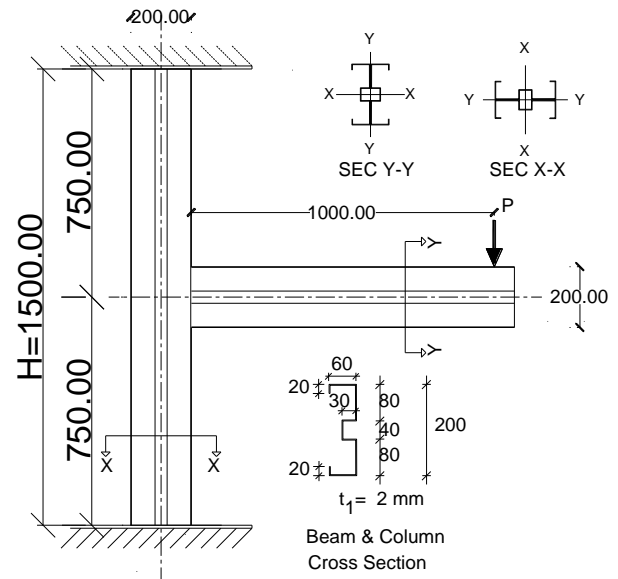
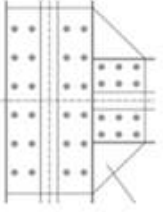
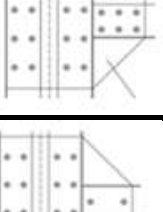
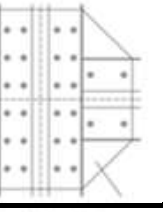
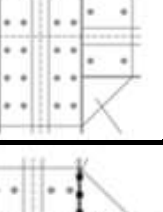
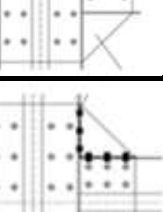
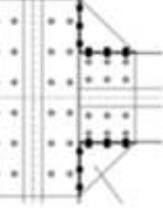
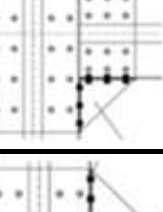
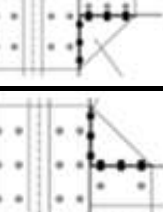
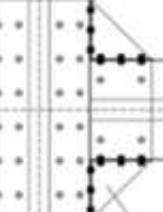
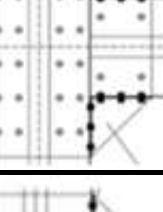
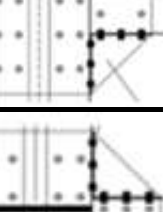
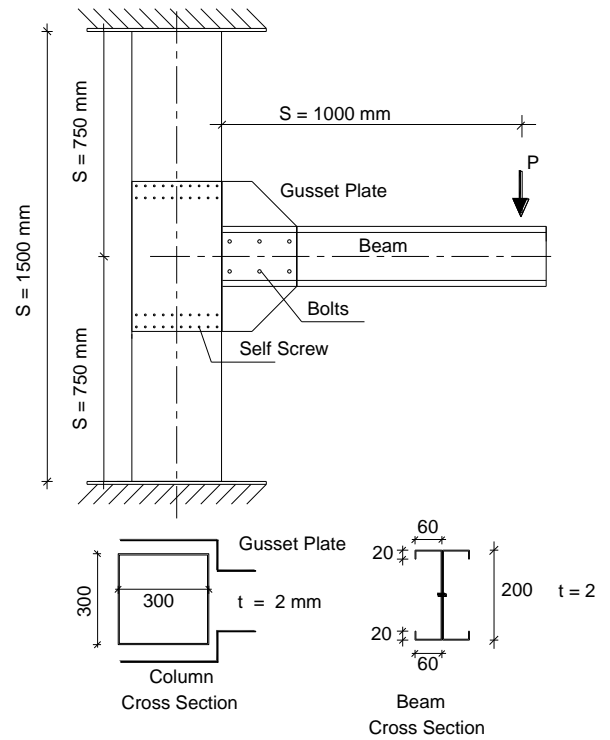


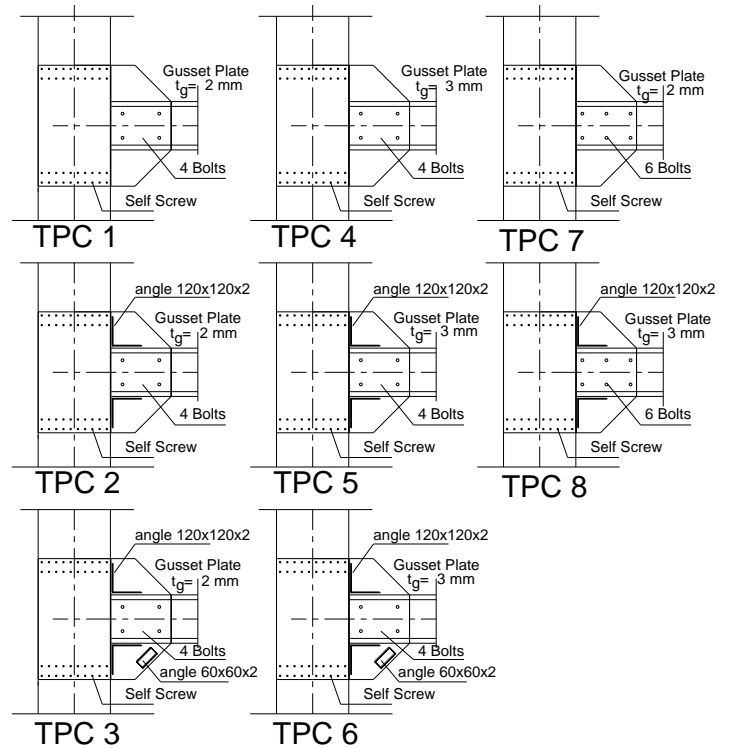
Figure 1: Case1 overall geometric dimensions.

Table 1 : Case1 connection details

No	Configuration	tg (mm)	Angle section
1		2	----
2		10	----
3		2	----
4		5	----
5		10	----
6		2	120X120X2
7		2	200X200X2
8		2	200X200X4
9		2	120X120X2
10		2	120X120X2
11		2	120X120X2



2-a) Case 2 overall geometric dimensions

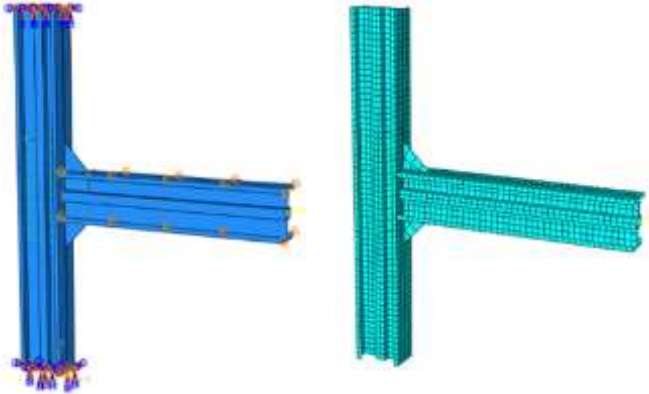


2-b) Case 2 connection details

Figure 2: Case2: connection.

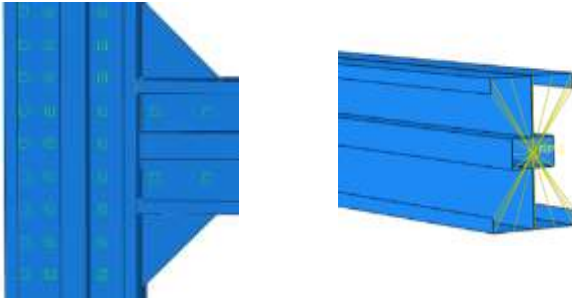
3. Finite Element Model

Finite element model was developed to simulate the behaviour of the studied connections. Finite element analysis package ABAQUS [8] was employed for the modeling and analysis procedure. The Shell S4R element was selected to model the connections. The mesh used was 10mm x 10mm. Nonlinear characteristics such as material nonlinearity, geometric nonlinearity were considered. The column two fixed ends are simulated by preventing translation of all nodes at the end sections along X, Y, and Z directions. To simulate the applied load a kinematic coupling constrained the 6 degrees of freedom was used at the edge surface of the beam by using a reference point at the C.G of the beam section and the displacement was applied at the reference point. Bolts are modelled as mesh independent point based deformable fastener and it is assigned with a beam connector section. The Young's modulus of elasticity and the yield stress of the steel used were considered as 210000MPa and 360 MPa, respectively for. The Possion's ratio was equal to 0.3. Bilinear stress strain curve was adopted. Newton-Raphson iterations were used in solving the nonlinear system of equation. Figure 3 illustrates the finite element models for case 1, and case2 connections.



a) Boundary Conditions

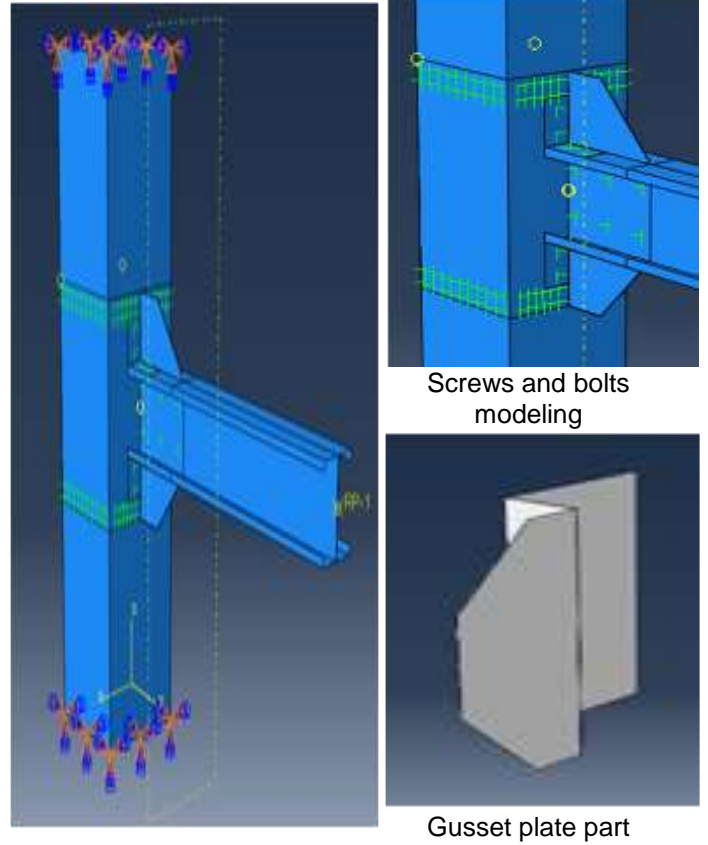
b) Mesh



Bolts Arrangements

Load applications

3-a) Case1: Columns and beams are Sigma sections



3-b) Case2: Columns are box and beams are lipped channel

Figure 3: Finite Element model

4. Results

Results are presented in the form of M-θ relationships, where, M, is the applied bending moments and θ, is the connection rotation. The rack manufacturing institute [9], Sarawit, A.T., Peköz, T, [10] introduce equations 1 & 2 to convert the load deflection curves to M-θ curves. Equation 1 represent the connection stiffness, F, where, where P is the applied vertical load on the cantilever and δ is the deflection of the cantilever. L_b: Length of Beam L_c: Length of Column I_c: Moment of Inertia of Column Cross Section I_b: Moment of Inertia of Beam Cross Section.

$$F = \frac{1}{\frac{\delta}{PL_b^2} - \frac{L_c}{16EI_c} - \frac{L_b}{3EI_b}} \quad (1)$$

$$F = \frac{M}{\theta} \quad (2)$$

The connection stiffness coefficient “F” and the rotational angle, θ , are determined from equations 1,2; respectively. Note, moment, M, also is determined by multiplying the load of cantilever by length of the beam.

To asses the efficiency of each connection configurations, the connection ultimate moments, M_a , are compared with the beam section ultimate capacity, M_c . The later is determined by a numerical finite element model for the beam. In this model (control model), the beam is simulated as cantilever element with fixed end. Note that, the beam section for case 1 is sigma back to back section, while for case 2 it is back to back lipped channel sections.

4.1 Case 1

The M- θ equilibrium paths of the 11 studied connections along with the control specimen are plotted in Figure 4. Results reflect that, the relationships are almost linear, however, it becomes nonlinear near the ultimate loads. Moreover, stiffness of the studied connections are flexible than the control case, since slope of the connection curves are smaller than the control one.

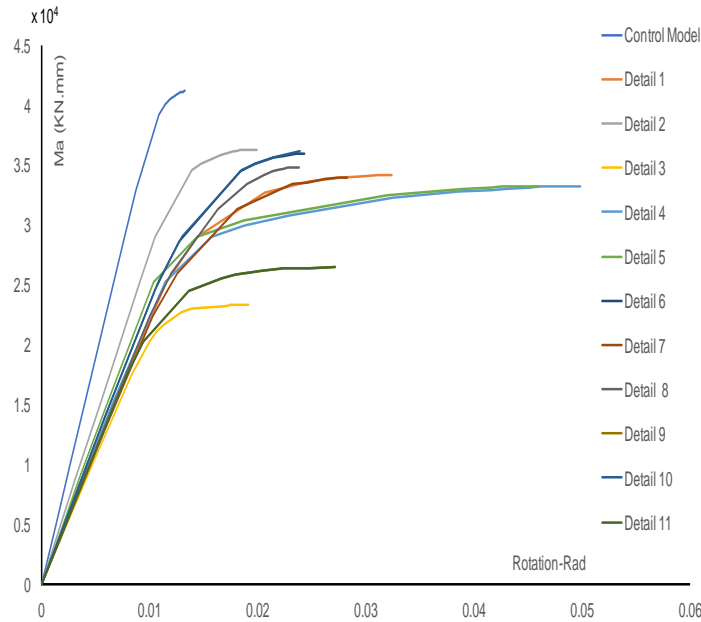
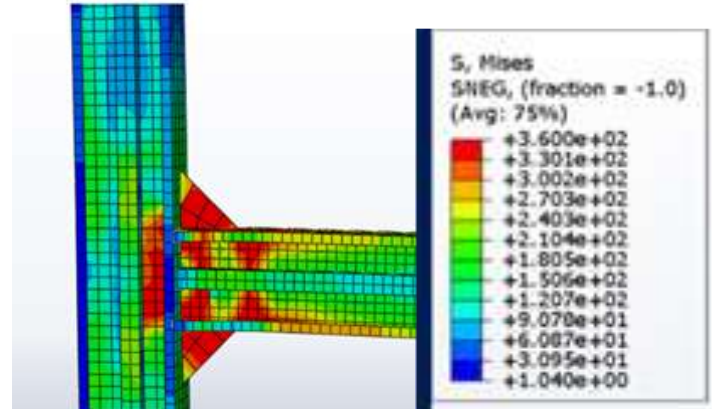


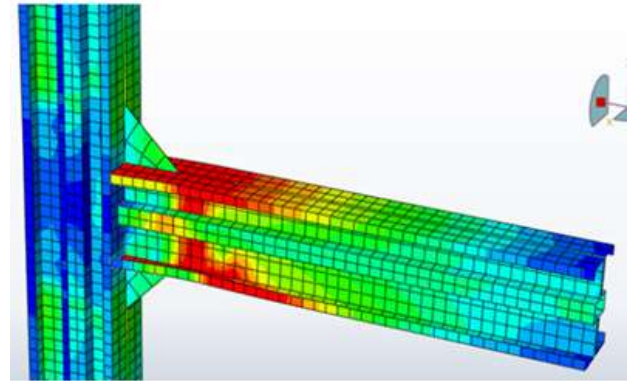
Figure 4: M- θ relationships for case 1 connections

The ultimate capacities of the studied connections range from 62% to 95% of the beam cross section capacity. Specimen 3 is the lowest capacity which reaches 60% of the beam cross sections. However, specimen 10 failed at moments equal to 95% of beam moments capacity. It is noticed that adding angles connecting beam to column flanges is considered the most effective parameter since the connection capacity increases from 62% to 93%. However, the connection capacity goes up from 62% to 91% when

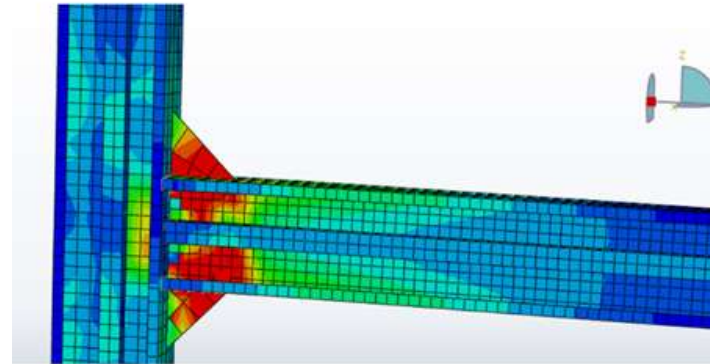
thickness of gusset plate increases from 2mm to 5mm. Similarly, when the number of bolts in the beam web increases from 4 to 6, the connection capacity changes from 62% to 92%.



Detail 1



Detail 2



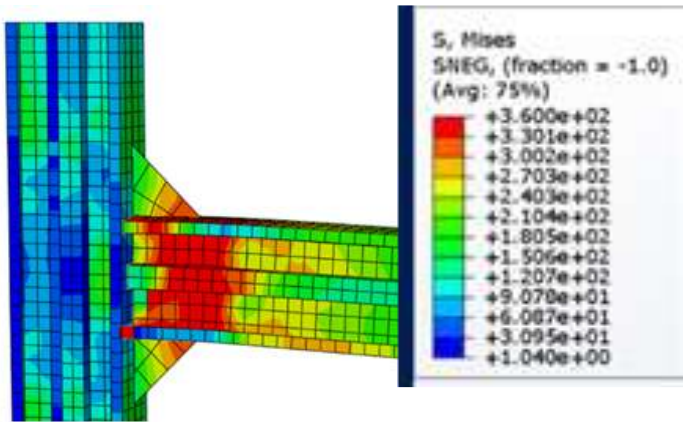
Detail 3

Figure 5: Failure modes and Von Mises stresses for details 1,2 and 3.

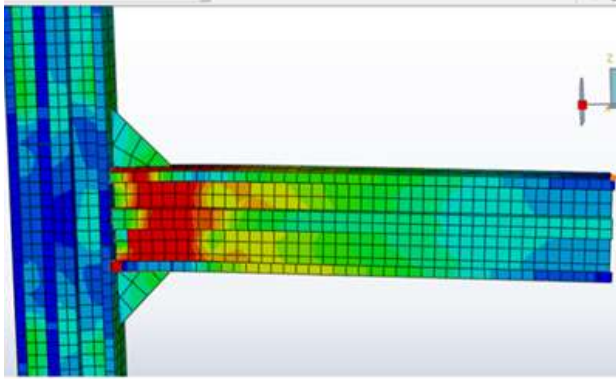
The Von Mises stresses distribution at failure moments are depicted in Figures 5 and 6 for details 1 to 6. It is clear that stresses reaches yield stresses around bolts between beam web and gusset plate. Moreover, increasing thickness of the

gusset plate, number of bolts, and adding stiffening angles between beam and column flanges pushes yielding to spread on the beam cross sections.

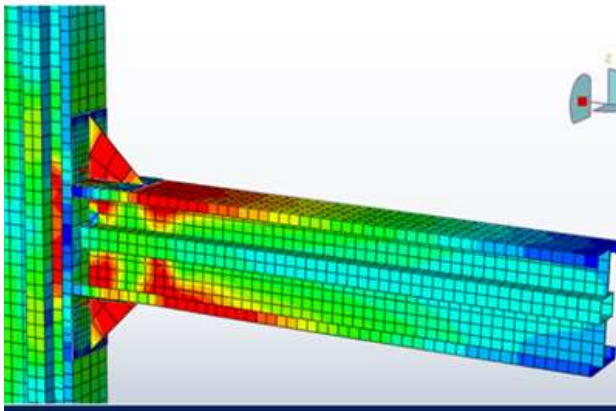
4.2 Case 2



Detail 4



Detail 5



Detail 6

Figure 6: Failure modes and Von Mises stresses for details 4,5, and 6.

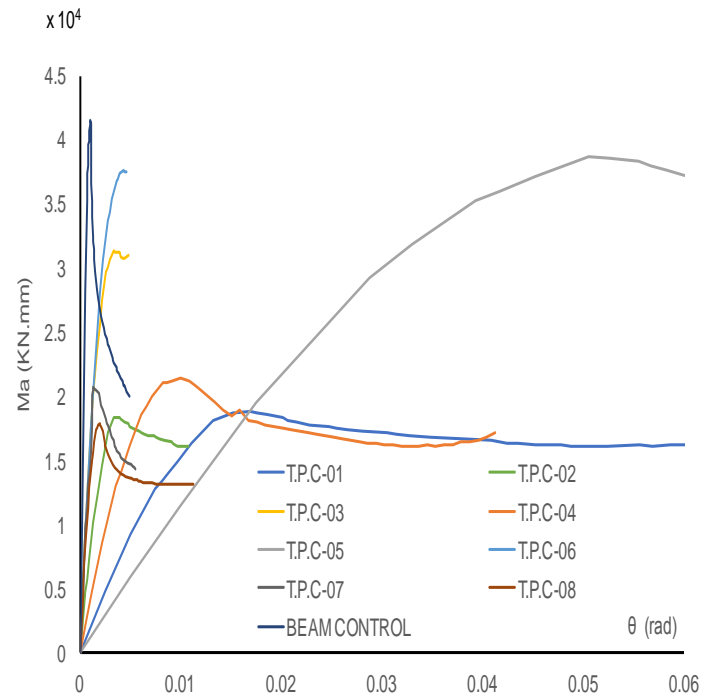


Figure 7: M- θ relationships for case 2 connections

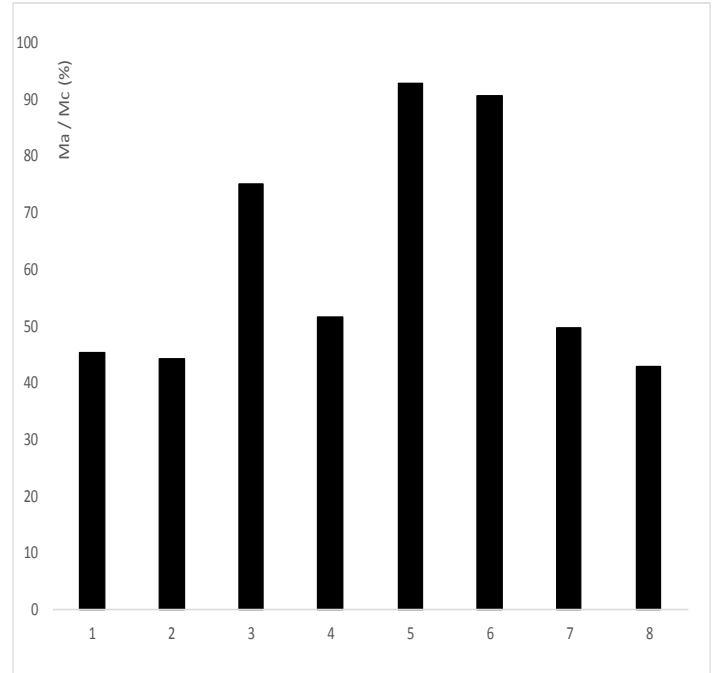


Figure 8: Strength ratios of the 8 details of connection case2 (Ma: connection capacity, Mc: control beam capacity)

Similarly, the ultimate moment capacity of the connections between back to back lipped channel beam sections and box column sections are drawn as function of the rotation, θ , in Figure 7. The M- θ curve of the control specimen is also added to the figure.

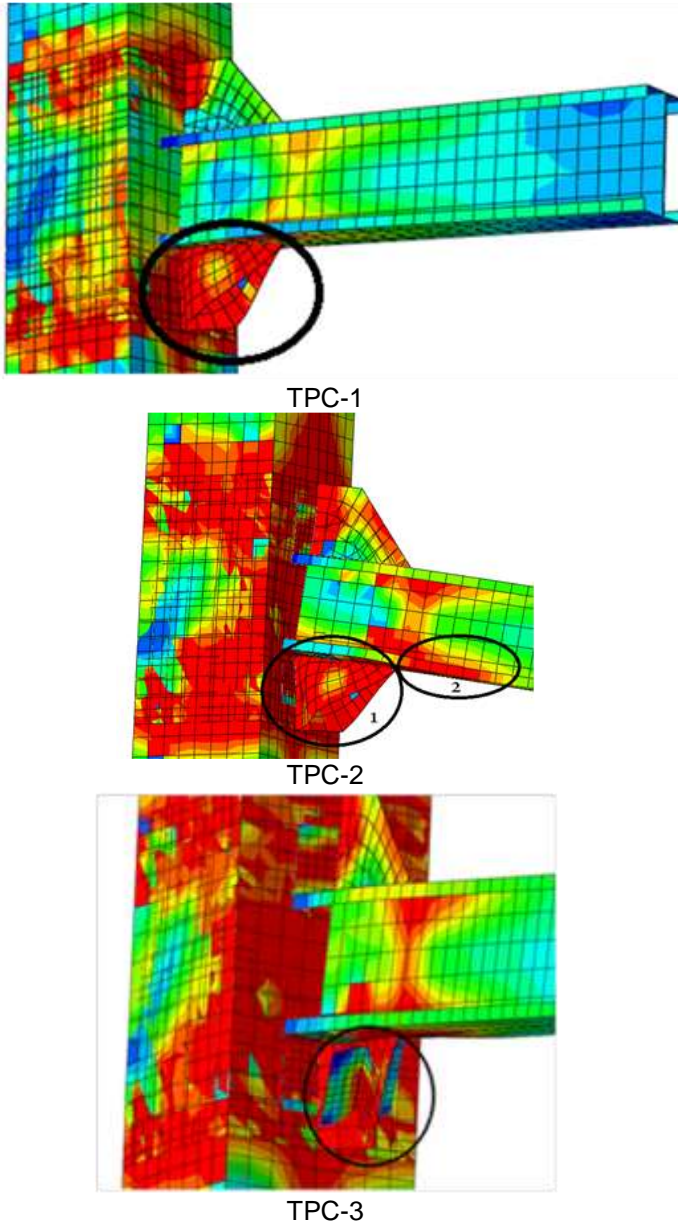


Figure 9: Failure mode in TPC-1.

Results clearly reflect that relations are nonlinear from the early stage of loading. In addition the details are flexible compared with the control beam. This is due to the local deformations that formed in the gusset plate. However, the connection stiffness increases significantly by adding diagonal angle in the compression part of the gusset plate. The connection ultimate strength ranges from 43% to 93%, Figure 8 illustrates these ratios. It is noticed that adding

stiffening angle between beam and column flanges in addition to diagonal angles in the compression part of the gusset plates are the sensitive parameters that increase the stiffness as well as strength of the connection details.

Figure 9 show the failure modes and Von-Misses stresses distribution of details 1,2, and 3. In detail 1, TPC-1, failure mainly due to local buckling in the compression part of the gusset plate. However, adding stiffening angles between beam and column flanges leads to spread of yielding in beam and column sections in addition to local buckling in the beam compression flange. Moreover, It is noticed that the yielded zones are concentrated in the beam and column sections

5. Conclusions

In this study the ultimate strength of two cases of beam to column moment connections are studied numerically by a nonlinear finite element model developed by ABAQUS software. In case 1, the beams and columns are formed of back to back sigma sections. However, in case 2, the columns are box sections and beams are back to back lipped channels. Results reveal that M- θ relationships for case 2 details are nonlinear from the early stage of loading compared with case 1 details. The ultimate strength of case 1 connections range between 62% to 95%, while for case 2 details it varied from 43% to 93%. For the two cases increasing thickness of gusset plate and adding stiffening angles between column and beam flanges significantly affect strength of the connections.

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